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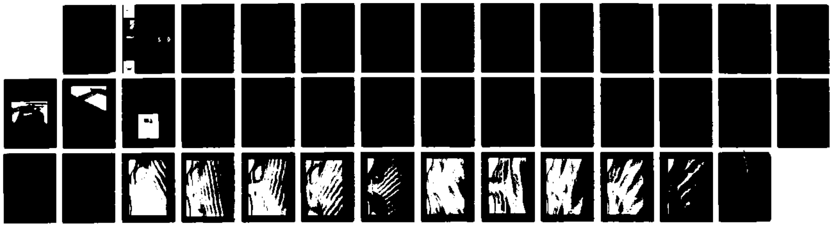

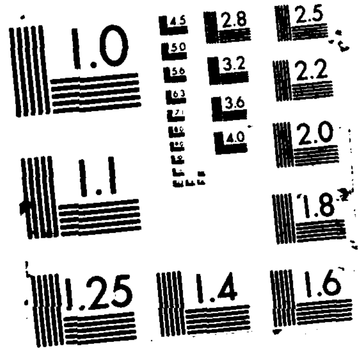
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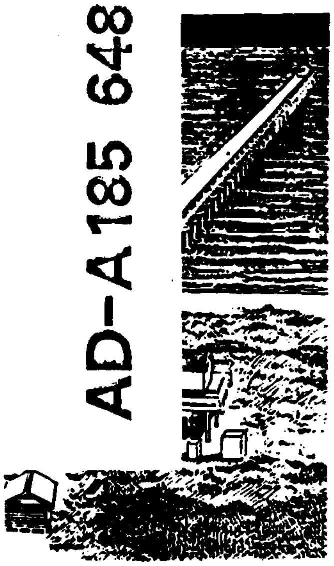

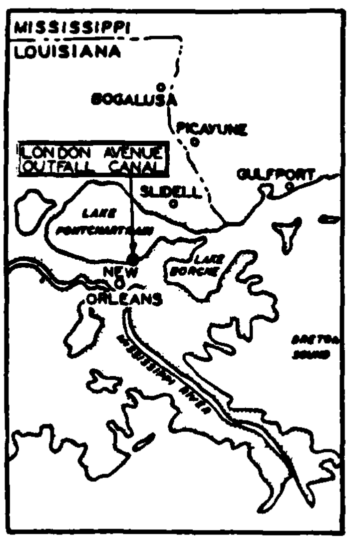

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EFFECTS OF WAVE ACTION ON A HURRICANE PROTECTION STRUCTURE FOR LONDON AVENUE OUTFALL CANAL, LAKE PONTCHARTRAIN NEW ORLEANS, LOUISIANA

Hydraulic Model Investigation

by

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Coastal Engineering Research Center

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PO Box 631, Vicksburg, Mississippi 39180-0631

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A 1:20-scale undistorted hydraulic model reproducing the lower reaches of the London Avenue Outfall Canal, the stepped seawall adjacent to Lakeshore Drive, and sufficient offshore area in Lake Pontchartrain to permit generation of the required test waves was used to investigate the effects of wave action at the site of a proposed hurricane protection structure. An 84-ft-long wave generator and an Automated Data Acquisition and Control System were utilized in model operation. Model tests indicated that wave conditions were very rough and turbulent along the lakefront and in the canal entrance (wave heights in excess of 9 ft) for hurricane storm wave conditions; however, wave heights in the vicinity of the proposed hurricane protection structure were negligible (less than 1 ft). The location of the proposed structure (in the lee of a bend in the canal) was satisfactory since it was not exposed to direct wave attack. The flat slopes of the overbank between the structure and the lake also expended wave energy as waves propagated up the canal.

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PREFACE

A request for a model investigation to evaluate the proposed location of a hurricane protection structure in the London Avenue Outfall Canal was initiated by the District Engineer, US Army Engineer District, New Orleans (LMN). Authorization for the US Army Engineer Waterways Experiment Station (WES) to perform the study was granted by the Office, Chief of Engineers, US Army Corps of Engineers. Funds were authorized by LMN on 14 May 1984 and 23 October 1985.

The model study was conducted during the period July 1984 through April 1986 at WES by personnel of the Hydraulics Laboratory (HL) and the Coastal Engineering Research Center (CERC). This report contains test results relative to wave conditions at the proposed hurricane protection structure which were conducted under the supervision of Dr. J. R. Houston; Chief, CERC; Mr. C. C. Calhoun, Jr., Assistant Chief of CERC; Mr. C. E. Chatham, Jr., Chief, Wave Dynamics Division; and Mr. D. G. Outlaw, Chief, Wave Processes Branch. The wave tests were conducted by Messrs. L. R. Tolliver and M. G. Mize, Civil Engineering Technicians, under the supervision of Mr. R. R. Bottin, Jr., Project Manager. This report was prepared by Messrs. Bottin and Mize. Test results involving details of the magnitude of the torque acting on the gates in the hurricane protection structure are reported in a WES technical report ("Hurricane Protection Structure for London Avenue Outfall Canal, Lake Pontchartrain, New Orleans, Louisiana") being prepared by J. R. Leech, HL, WES.

The authors wish to acknowledge Messrs. L. Cook, R. Louque, E. Walker, and F. Weaver of the US Army Engineer Division, Lower Mississippi Valley; and COL E. S. Witherspoon; Messrs. F. Chatry, C. Soileau, R. Guizerix, V. Stutts, J. Combe, T. Hassenboehler, and D. Strecker; and Ms. J. Hote of LMN who visited WES to observe model operation and participate in conferences during the course of the model study. This report was edited by Ms. S. A. J. Hanshaw, Information Products Division, Information Technology Laboratory, WES.

COL D. G. Lee, CE, was Commander and Director of WES during the preparation and publication of the report. Dr. R. W. Whalin was Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
cubic feet per second	0.2831685	cubic metres per second
feet	0.3048	metres
gallons	3.785	litres
inches	0.0254	metres
miles (US statute)	1.609347	kilometres
square feet	0.09290304	square metres
square miles (US statute)	2.589998	square kilometres

EFFECTS OF WAVE ACTION ON A HURRICANE PROTECTION STRUCTURE FOR LONDON AVENUE OUTFALL CANAL, LAKE PONTCHARTRAIN, NEW ORLEANS, LOUISIANA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. New Orleans, Louisiana, has a unique drainage system that includes 18 pumping stations on the east bank of the Mississippi River and two on the west bank. These stations have a combined capacity of 25 billion gallons* per day (enough to empty a lake of 10 square miles, 11 ft deep, in 24 hours). The city's average annual rainfall is 58.12 in., exceeded only by two other metropolitan areas in the United States: Miami, Florida, and Mobile, Alabama. Approximately 55,085 acres in the developed portion of New Orleans and 2,640 acres in adjoining Jefferson Parish require drainage to prevent flooding.

2. During periods of dry weather small amounts of water are diverted to sewage pumping stations for discharge into the river. During heavy rains the large drainage pumps go into operation discharging storm water into lake-level open channels leading to Lake Pontchartrain or Lake Borgne via Bayou Bienvenue. A vicinity map is shown in Figure 1.

3. The London Avenue Outfall Canal is among three canals being considered for hurricane surge protection on the south shore of Lake Pontchartrain (Figure 2). The primary purpose of the outfall canal is to transport interior drainage from part of New Orleans to Lake Pontchartrain. A pumping station with a capacity of 8,000 cfs is located at the origin of the canal, approximately 3 miles south of the lake front. Parallel levees from the lake front to the pumping station have elevations** of +10 ft, and the levee along the lake front has an el of +15 ft.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to National Geodetic Vertical Datum (NGVD) of 1929.

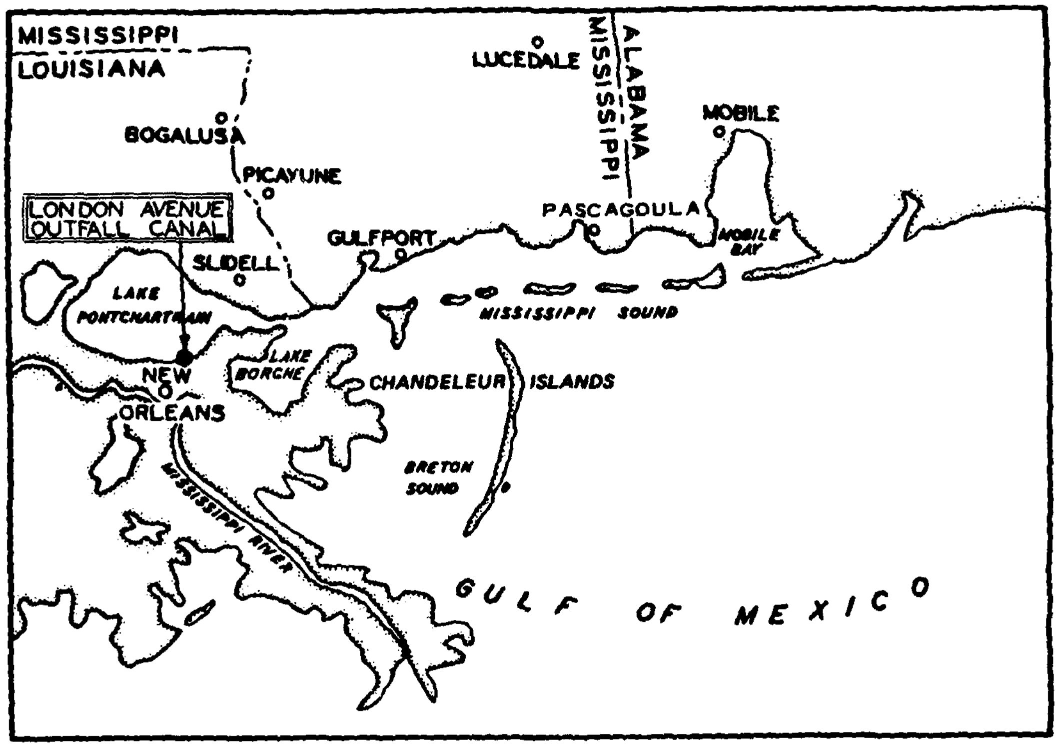

Figure 1. Vicinity map

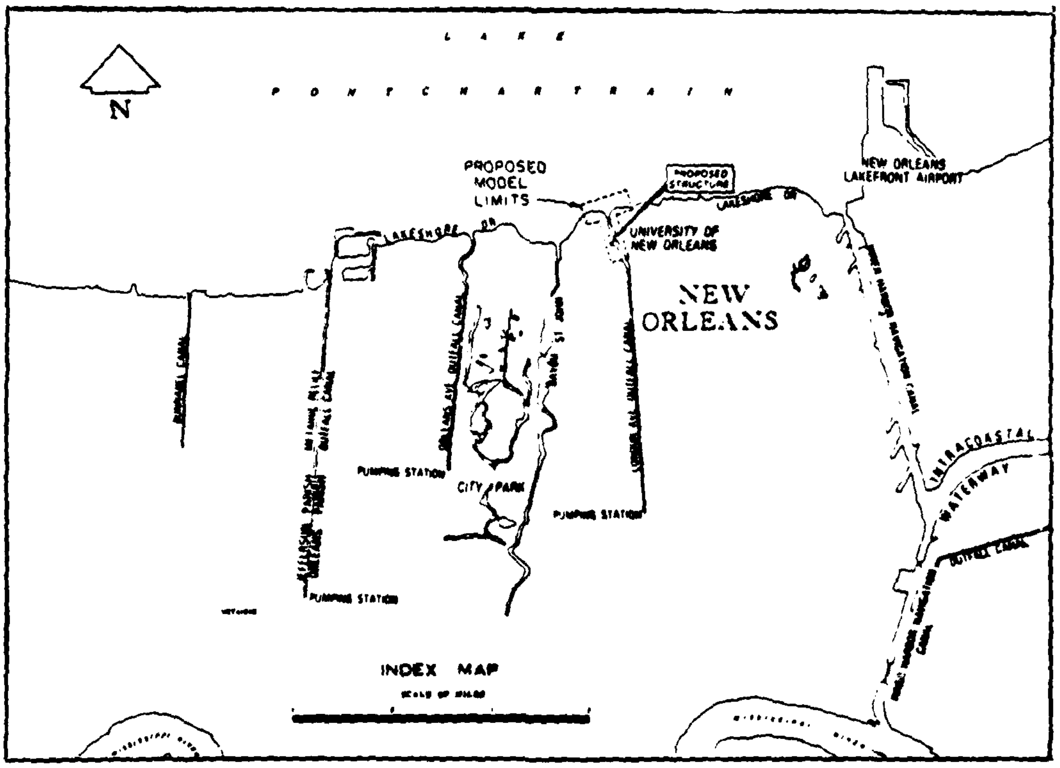

Figure 2. Location of London Avenue Outfall Canal

Proposed Improvements

4. The existing levee system is inadequate in providing flood protection to the City of New Orleans for a 100-year hurricane storm surge. A plan, therefore, is proposed to provide hurricane protection to the City. The proposed plan consists of raising the levees to an el of +18 ft along the lake front and then tapering the levee from the +18-ft el to a +14-ft el along both sides of the canal to a point approximately 1,000 ft southerly where a gated hurricane protection structure would be installed across the outfall canal. The proposed hurricane protection structure design is based on the theory of self-opening and closing, vertical, eccentrically pinned butterfly gates. The butterfly gates would remain open during pumping of the interior drainage into the lake as long as the water level in the outfall canal exceeded that on the lake side of the structure (Figure 2). During an incoming surge, when the water level would be greater on the lake side than on the pumping station side of the outfall canal, the gates would automatically close. This concept would permit continuous operation of the pumping station during a hurricane and automatic reopening of the gates when the water level in the outfall canal downstream of the pumping station exceeded that on the lake side of the control structure.

Purpose of the Investigation

5. At the request of the US Army Engineer District, New Orleans (LMN), a hydraulic model investigation was initiated by the US Army Engineer Waterways Experiment Station (WES) to evaluate the proposed location of the hurricane protection structure and to develop a gate and canal design that would permit automatic flow-induced opening and closing of the gates when subjected, respectively, to pumped flows or hurricane surges. The investigation was conducted jointly by the Hydraulics Laboratory (HL) and the Coastal Engineering Research Center (CERC) at WES. This report presents test results relative to the wave climate at the proposed hurricane protection structure. Test results involving the automatic flow-induced opening and closing of the butterfly gates and details of the torque magnitudes acting on the vertical gate shafts have been compiled by HL personnel and are detailed in a WES technical report by Leech (in preparation).

PART II: THE MODEL

Design of the Model

6. The London Avenue Outfall Canal model (Figure 3) was constructed to an undistorted linear scale of 1:20, model to prototype. Scale selection was based on such factors as

- a. Depth of water required in the model to prevent excessive bottom friction.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Available wave-generating and wave-measuring equipment.
- f. Model construction costs.
- g. The requirement to simulate flow conditions and forces at the gated structure.

A geometrically undistorted model was necessary to ensure accurate reproduction of short-period wave and current patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law (American Society of Civil Engineers 1942). The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Model:Prototype Scale Relation
Length	L	$L_r = 1:20$
Area	L^2	$A_r = L_r^2 = 1:400$
Volume	L^3	$V_r = L_r^3 = 1:8,000$
Time	T	$T_r = L_r^{1/2} = 1:4.47$
Velocity	L/T	$V_r = L_r^{1/2} = 1:4.47$

* Dimensions are in terms of length and time.

The Model and Appurtenances

7. The model, which was molded in cement mortar, reproduced about 3,000 ft of the lower reach of the London Avenue Outfall Canal; approximately

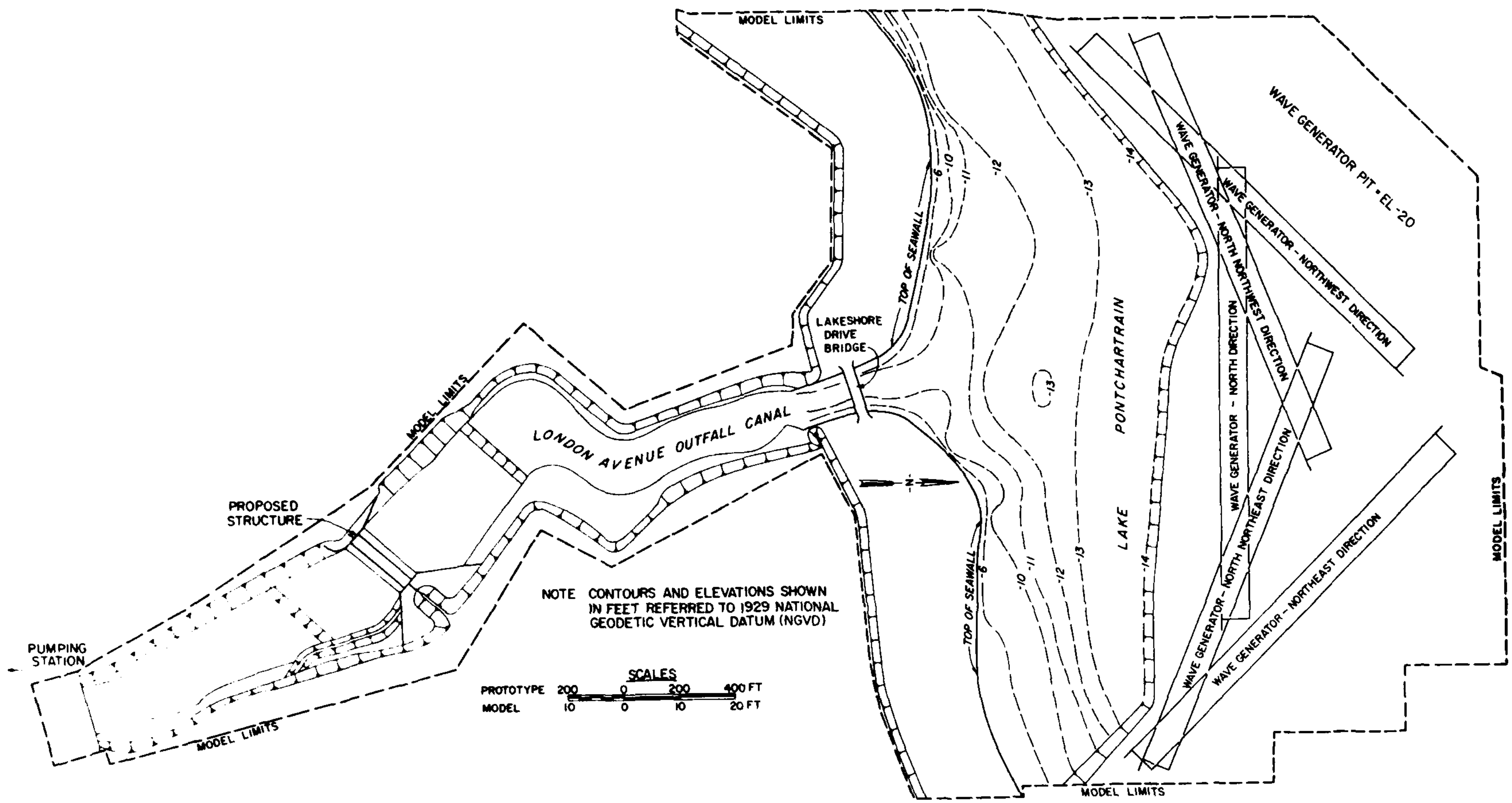

Figure 3. Model layout

1,500 ft and 1,300 ft of the stepped seawall shoreline to the east and west, respectively, of the canal entrance; the control structure; the Lakeshore Drive Bridge at the canal entrance; and underwater contours in Lake Pontchartrain to an offshore depth of -14 ft (with a sloping transition to the wave generator pit el of -20 ft). The total area reproduced in the model was approximately 18,500 sq ft, representing about 0.3 square miles in the prototype. A general view of the model (looking lakeward) is shown in Figure 4, and a view of the Lakeshore Drive Bridge is shown in Figure 5. Vertical control for model construction was referenced to the NGVD of 1929. Horizontal control was referenced to a local prototype grid system.

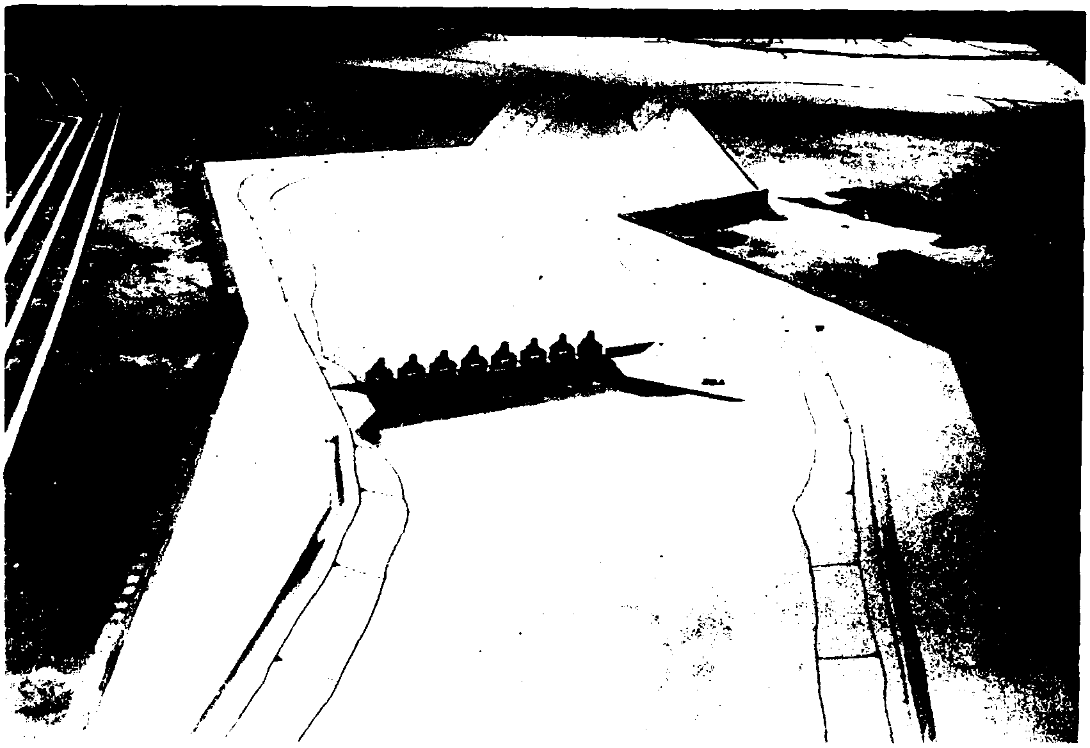

Figure 4. General view of model

8. Model waves were generated by an 84-ft-long piston-type wave generator. The horizontal movement of the piston plate caused a periodic displacement of water incident to this motion. The length of the stroke and the frequency of the piston plate movement were variable over the range necessary to generate waves with the required characteristics. In addition, the wave generator was mounted on retractable casters which enabled it to be positioned to generate waves from the required directions.

9. An Automated Data Acquisition and Control System (ADACS), designed

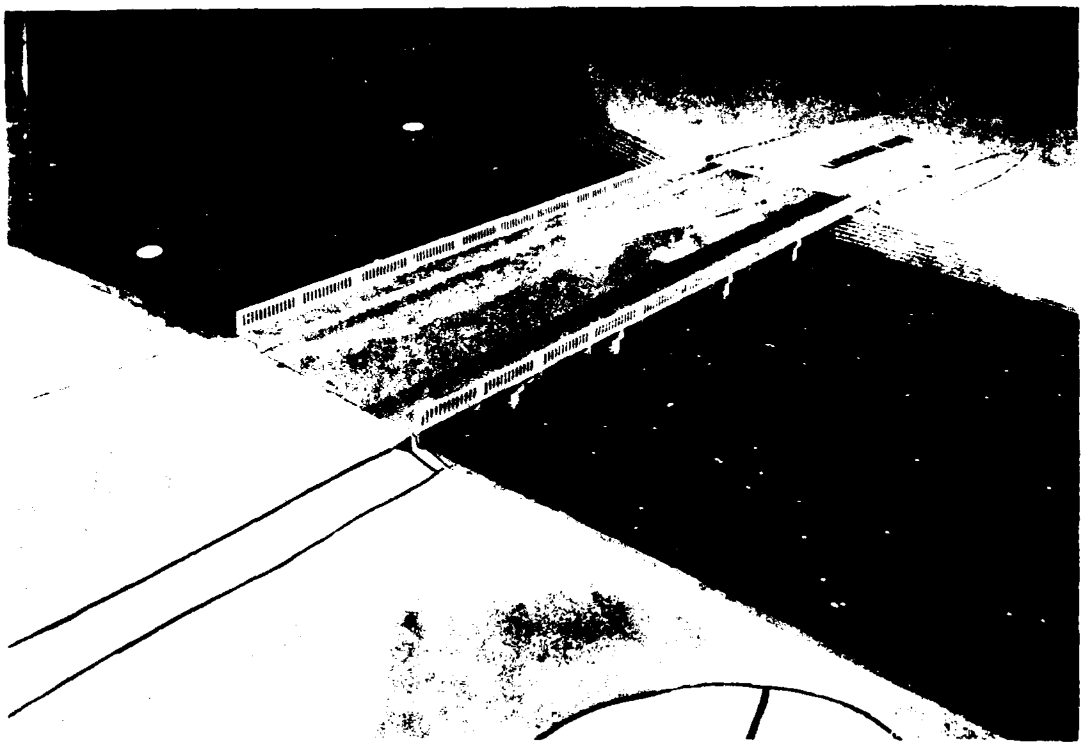

Figure 5. View of Lakeshore Drive Bridge (looking northeast)

and constructed by WES (Figure 6), was used to secure wave height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic tape the electrical output of parallel-rod, resistance-type sensors (Figure 7) that measured the change in water-surface elevation with respect to time. The magnetic tape output then was analyzed to obtain the required data.

10. Guide vanes were placed along the wave generator sides in the flat pit area to ensure proper formation of the wave train incident to the model contours. In addition, a 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to dampen any wave energy that might otherwise be reflected from the model walls.

11. A water circulation system was used in the model to reproduce hurricane surge conditions and the pumping of drainage water from the City of New Orleans into Lake Pontchartrain. Details of this system can be obtained from Leech (1987).

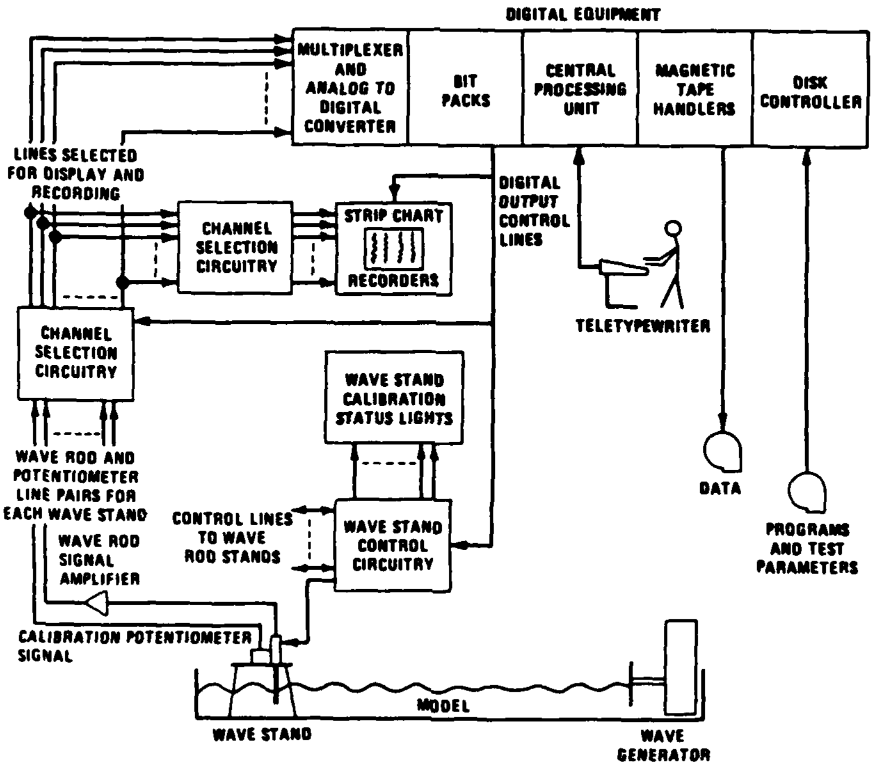

Figure 6. Automated Data Acquisition and Control System

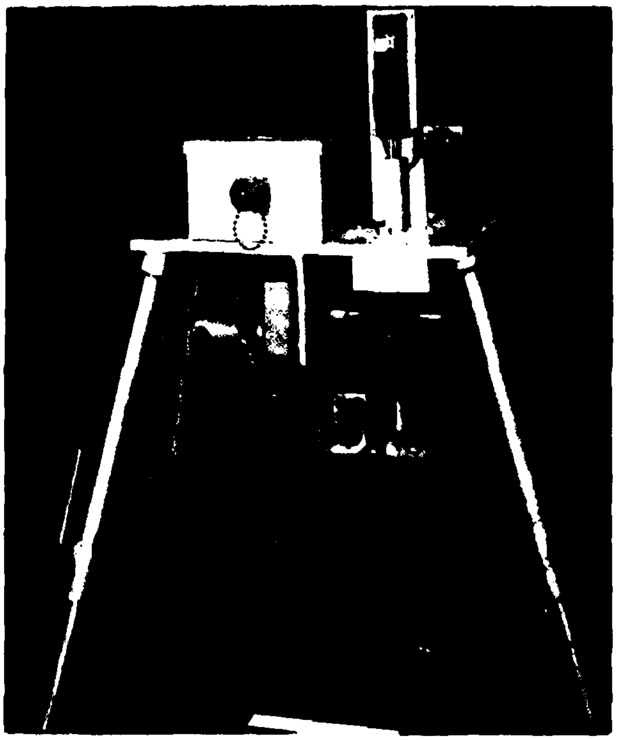

Figure 7. Parallel-rod wave sensor

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

12. Still-water levels (swl's) for wave action models are selected so that the various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include refraction of waves in the project area, overtopping of structures by the waves, and reflection of wave energy from various structures. In most cases it is desirable to select a model swl that closely approximates the higher water stages which normally occur in the prototype for the following reasons:

- a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle.
- b. Most storms moving onshore are characteristically accompanied by a higher water level because of wind tide and shoreward mass transport.
- c. The selection of a high swl helps minimize model scale effects resulting from viscous bottom friction.
- d. When a high swl is selected, a model investigation tends to yield more conservative results.

13. Swl's of 0.0, +5.0, and +11.5 ft (NGVD) were selected by LMN for use during model testing. The lower value (0.0 ft) represents normal lake conditions; the +5.0-ft swl represents high water conditions that occur annually; and the higher value (+11.5 ft) represents surge conditions associated with a Standard Project Hurricane (SPH). The SPH represents a hurricane with a 100-year recurrence interval.

Factors influencing selection of test wave characteristics

14. In planning the testing program for a model investigation of wave action problems, it is necessary to select dimensions and directions for the test waves that will allow a realistic test and an accurate evaluation of proposed conditions. Surface wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the length of time

that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Selection of test wave conditions entails evaluation of such factors as

a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generation area) for various directions from which waves can attack the problem area.

b. The frequency of occurrence and duration of storm winds from the various directions.

c. The alignment, size, and relative geographic position of the entrance.

d. The alignments, lengths, and locations of the various reflecting surfaces.

e. The refraction of waves caused by differentials in depth in the area lakeward of the site which may create either a concentration or a diffusion of wave energy.

Wave refraction

15. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to the selection of test wave characteristics are the changes resulting from wave refraction and shoaling. The change in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. The shoaling coefficient (a function of wave length and water depth) can be obtained from the Shore Protection Manual (SPM) (1984). Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transfer of deepwater wave heights to shallow-water values.

16. Because of the limited depth in Lake Pontchartrain (-14 ft) and the limited fetch (20 miles), a wave refraction analysis was not conducted for the London Avenue Outfall Canal site. The magnitude and direction of winds approaching the area from over the Lake Pontchartrain water body were considered to be the governing factors, and all waves were assumed to be locally generated. For this study, critical directions of wave approach were determined to be from northwest, north-northwest, north, north-northeast, and northeast.

Selection of test waves

17. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the London

Avenue Outfall Canal. However, statistical wave hindcast data representative of this area were obtained from a study performed previously at WES (Bottin and Turner 1980) at the Seabrook Lock Complex (located approximately 2 miles east of the London Avenue Canal). This study provided wave data for the north, north-northwest, and northwest directions. Wave data from the more easterly directions (north-northeast and northeast) were obtained by the application of hindcasting techniques from Vincent and Lockhart (1983) to wind data acquired at the New Orleans International Airport. The following tabulation shows selected test directions and the corresponding test wave characteristics and swl's which were utilized in model operation.

	Test Waves		
Direction	Period sec	Height ft	swl's
Northwest	3	2	0.0, +5.0, +11.5
		4	0.0, +5.0, +11.5
	4	4	0.0, +5.0, +11.5
	7.3*	7.8*	+11.5
North-northwest	3	2	0.0, +5.0, +11.5
		4	0.0, +5.0, +11.5
	4	4	0.0, +5.0, +11.5
	7.3*	7.8*	+11.5
North	3	2	0.0, +5.0, +11.5
		4	0.0, +5.0, +11.5
	4	5	0.0, +5.0, +11.5
	7.3*	7.8*	+11.5
North-northeast	3	2	0.0, +5.0, +11.5
		4	0.0, +5.0, +11.5
	4	4	0.0, +5.0, +11.5
	7.3*	7.8*	+11.5
Northeast	3	2	0.0, +5.0, +11.5
		4	0.0, +5.0, +11.5
	4	4	0.0, +5.0, +11.5
	7.3*	7.8*	+11.5

* Wave characteristics provided by LMN associated with an SPH.

Analysis of Model Data

18. In the wave height data analysis, the average height of the highest one-third of the waves recorded at each gage location (significant wave height) was computed. All wave heights then were adjusted to compensate for

excessive model wave height attenuation resulting from viscous bottom friction by application of Keulegan's equation (Keulegan 1950). From this equation reduction of wave heights in the model (relative to the prototype) can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel.

PART IV: TESTS AND RESULTS

Tests

19. Prior to conducting wave height tests in the London Avenue Outfall Canal, the original canal alignment and excavation both upstream and downstream of the control structure was accomplished to improve flow distribution in the approach and through the structure, according to Leech (in preparation). After the canal design was optimized, wave height data were obtained in the lower reaches of the canal and adjacent to the hurricane protection structure. Wave gage locations are shown in Figure 8. Wave heights were obtained with the gates of the structure in the open position in order to minimize standing waves which may result from wave reflection off the structure. Wave height tests were conducted for all test conditions from all five wave directions. For hurricane conditions tests were conducted also with the Lakeshore Drive Bridge superstructure removed which would represent destruction of this structure. Wave pattern photographs were secured for representative test waves from all five directions.

Results

20. Results of wave height tests with the 0.0-ft swl are presented in Table 1. Maximum wave heights were 5.9 ft in the canal entrance (Gage 1) for 4-sec, 5-ft test waves from north; 3.7 ft in the canal south of the Lakeshore Drive bridge (Gage 2) for 4-sec, 5-ft test waves from north; and 0.1 ft at the bend in the canal (Gage 6) for several test waves from north and north-northwest. Wave heights were <0.1 ft adjacent to the proposed structure (Gages 8-10) for all the test waves.

21. Wave height measurements obtained with the +5.0-ft swl are presented in Table 2. Maximum wave heights were 5.8 ft in the canal entrance (Gage 1) for 3-sec, 4-ft and 4-sec, 4-ft test waves from north-northeast; 3.9 ft in the canal south of the bridge (Gage 2) for 3-sec, 4-ft test waves from north-northwest; 0.3 ft at the bend in the canal (Gage 6) for test waves from northwest, north-northwest, and north; and 0.2 ft adjacent to the proposed structure (Gages 8-10) for test waves from north-northwest and north. Waves were observed breaking over the stepped seawall adjacent to the lake.

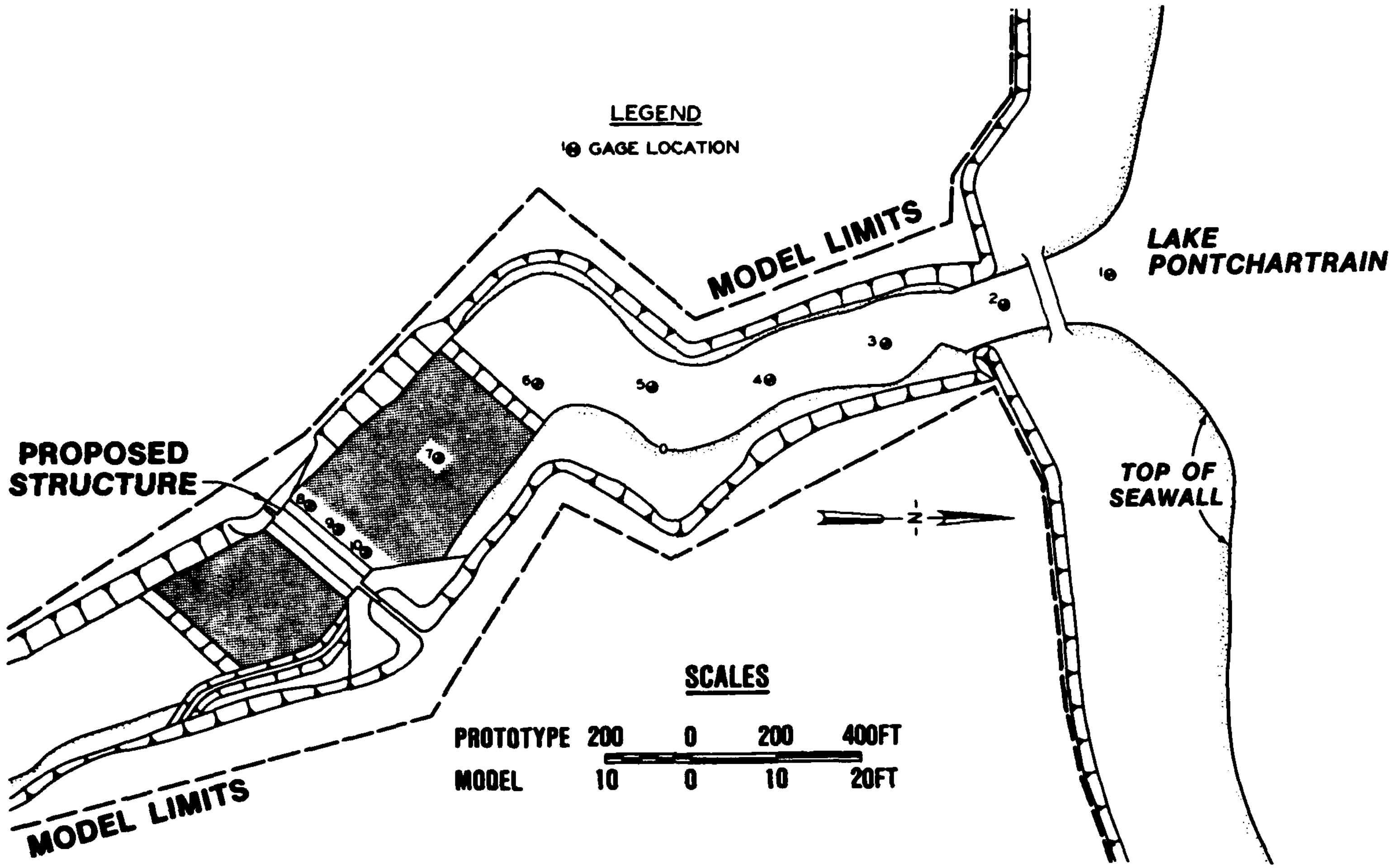

Figure 8. Wave gage locations for London Avenue Outfall Canal

Typical wave patterns at the canal entrance for the +5-ft swl are shown in Photos 1-5.

22. Wave heights obtained for the +11.5-ft swl (with the bridge in place) are presented in Table 3. Maximum wave heights were 9.2 ft in the canal entrance (Gage 1); 3.0 in the canal south of the bridge (Gage 2); 1.0 ft at the bend in the canal (Gage 6); and 0.8 ft adjacent to the proposed structure (Gage 9) all for 7.3-sec, 7.8-ft test waves from north-northwest. Waves overtopped the stepped seawall adjacent to the lake and flooded the area between the lake and the levee. Typical wave patterns obtained at the canal entrance for the +11.5-ft swl are shown in Photos 6-10.

23. Wave height data secured for the +11.5-ft swl with the bridge superstructure removed are shown in Table 4. Maximum wave heights were 9.3 ft in the canal entrance (Gage 1) and 3.9 ft in the canal south of the bridge (Gage 2) for 7.3-sec, 7.8-ft test waves from north-northwest; 0.9 ft at the bend in the canal (Gage 6) for 7.3-sec, 7.8-ft test waves from northwest and north-northwest; and 0.7 ft adjacent to the proposed structure (Gage 8) for 7.3-sec, 7.8-ft test waves from north.

Discussion of Test Results

24. Test results indicated very rough and turbulent wave activity at the canal entrance for storm wave conditions. Wave energy dissipated quickly, however, as waves propagated up the canal. Based on visual observations the existing slopes of the canal banks (south of the seawall) served as an excellent dissipater where waves expended energy as they refracted up the slopes. The bend in the canal (north of the proposed structure) was also instrumental in reducing the amount of wave energy reaching the structure.

25. The maximum wave height obtained adjacent to the proposed structure was 0.8 ft, which occurred for the SPH at one gage location for one direction. For normal storm wave conditions, wave heights did not exceed 0.2 ft adjacent to the structure. Considering all test conditions, wave heights adjacent to the proposed hurricane protection structure were considered negligible. The location of the proposed structure in the lee of the bend in the canal was considered to be satisfactory. It should be noted, however, that this is a site-specific study, and the application of these results to a structure with more direct exposure to incoming wave activity may not be valid.

PART V: CONCLUSIONS

26. Although wave conditions were very rough and turbulent along the lakefront and in the canal entrance (wave heights in excess of 9 ft), for hurricane storm wave conditions wave heights in the vicinity of the proposed hurricane protection structure were negligible (less than 1 ft). The location of the proposed structure (in the lee of a bend in the canal) was satisfactory since it was not exposed to direct wave attack. The flat slopes of the overbank between the structure and the lake also expended wave energy as waves propagated up the canal.

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Table 1

Wave Heights Obtained in the London Avenue Outfall Canal for the 0.0-ft swl

Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, in ft, at Indicated Gage Location: Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
Northwest	3	2	1.5	1.3	0.2	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
		4	5.1	1.1	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	4	4	5.1	1.6	0.7	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
North-Northwest	3	2	3.0	2.1	0.9	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1
		4	5.0	2.8	0.7	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	4	4	5.4	2.6	0.8	0.6	0.2	0.1	<0.1	<0.1	<0.1	<0.1
North	3	2	2.3	0.9	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
		4	5.4	2.9	0.8	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	4	5	5.9	3.7	1.0	0.6	0.2	0.1	<0.1	<0.1	<0.1	<0.1
North-Northeast	3	2	2.9	0.9	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
		4	4.5	0.9	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	4	4	5.1	1.4	0.4	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Northeast	3	2	1.5	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
		4	3.9	1.1	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	4	4	4.1	0.6	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1

Table 2

Wave Heights Obtained in the London Avenue Outfall Canal for the +5.0-ft swl

Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, in ft, at Indicated Gage Location: Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
Northwest	3	2	2.1	0.9	0.5	0.7	0.3	0.2	0.1	0.1	0.1	0.1
		4	5.5	2.1	0.8	0.4	0.3	0.2	0.1	0.1	0.1	0.1
	4	4	5.6	3.4	1.4	0.9	0.6	0.3	0.2	0.1	0.1	0.1
North-Northwest	3	2	2.9	1.8	1.4	1.0	0.8	0.2	0.1	0.1	0.1	0.1
		4	4.1	3.9	1.7	1.3	0.8	0.3	0.2	0.1	0.1	0.1
	4	4	5.3	3.8	1.6	1.0	0.6	0.3	0.2	0.2	0.2	0.2
North	3	2	2.8	1.7	2.1	1.3	0.8	0.2	0.1	0.2	0.1	0.2
		4	4.8	3.4	1.6	1.0	0.6	0.2	0.1	0.1	0.1	0.1
	4	5	4.1	1.8	1.0	1.0	0.6	0.3	0.2	0.2	0.2	0.2
North-Northeast	3	2	2.6	1.5	0.6	0.3	0.1	<0.1	<0.1	<0.1	<0.1	0.1
		4	5.8	1.9	0.9	0.5	0.2	0.1	0.1	0.1	0.1	0.1
	4	4	5.8	3.4	0.6	0.6	0.2	0.2	0.1	0.1	0.1	0.1
Northeast	3	2	2.2	0.6	0.1	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
		4	3.7	0.5	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	4	4	4.6	1.0	0.2	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1

Table 3

Wave Heights Obtained in the London Avenue Outfall Canal for the +11.5-ft swl

Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, in ft, at Indicated Gage Location: Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
Northwest	3	2	2.4	0.2	0.2	0.1	0.1	0.1	<0.1	<0.1	<0.1	<0.1
		4	4.7	0.7	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1
	4	4	4.4	0.6	0.2	0.4	0.2	0.1	0.1	0.1	0.1	0.1
	7.3	7.8	6.4	2.0	1.5	1.6	1.0	0.8	0.6	0.5	0.4	0.3
North-Northwest	3	2	1.8	0.4	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1
		4	4.7	0.8	0.3	0.3	0.2	0.1	0.1	0.1	0.1	0.1
	4	4	3.7	1.1	0.4	0.5	0.4	0.2	0.2	0.1	0.1	0.1
	7.3	7.8	9.2	3.0	1.7	1.6	0.9	1.0	0.5	0.5	0.8	0.6
North	3	2	3.8	0.5	0.5	0.3	0.1	0.2	0.1	0.1	0.1	0.1
		4	5.2	1.0	0.4	0.4	0.2	0.2	0.2	0.1	0.1	0.2
	4	5	5.3	1.0	0.5	0.7	0.5	0.3	0.2	0.2	0.1	0.2
	7.3	7.8	6.1	1.8	1.5	1.2	0.9	0.4	0.4	0.4	0.4	0.4
North-Northeast	3	2	1.9	1.4	0.7	0.7	0.2	0.3	0.2	0.2	0.2	0.2
		4	5.2	1.3	0.9	0.7	0.3	0.3	0.3	0.2	0.2	0.2
	4	4	3.9	1.6	0.8	0.7	0.4	0.3	0.3	0.2	0.3	0.3
	7.3	7.8	7.8	2.2	1.4	1.2	1.0	0.7	0.5	0.5	0.5	0.5
Northeast	3	2	1.3	0.9	0.8	0.5	0.1	0.3	0.2	0.3	0.2	0.2
		4	4.1	1.1	0.9	0.5	0.4	0.3	0.2	0.1	0.3	0.2
	4	4	3.2	0.7	0.9	0.8	0.9	0.7	0.3	0.4	0.4	0.4
	7.3	7.8	5.6	1.6	1.4	1.2	0.9	0.6	0.5	0.4	0.5	0.5

Table 4

Wave Heights Obtained in the London Avenue Outfall Canal for the +11.5-ft swl with the Bridge Superstructure Removed

Direction	Test Wave		Wave Height, in ft, at Indicated Gage Location									
	Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
Northwest	3	2	3.1	0.9	0.7	0.5	0.4	0.2	0.2	0.1	0.2	0.1
		4	4.4	1.3	0.8	0.6	0.4	0.2	0.2	0.2	0.1	0.2
	4	4	4.2	1.4	0.8	0.5	0.2	0.3	0.3	0.2	0.2	0.2
	7.3	7.8	8.1	3.6	2.0	2.0	1.0	0.9	0.7	0.6	0.5	0.6
North-Northwest	3	2	1.9	1.7	1.1	1.0	0.4	0.3	0.2	0.2	0.4	0.3
		4	4.0	2.3	1.6	0.8	0.6	0.4	0.5	0.2	0.3	0.3
	4	4	4.1	2.4	1.0	0.8	0.7	0.4	0.4	0.4	0.4	0.3
	7.3	7.8	9.3	3.9	2.5	2.1	1.3	0.9	0.8	0.6	0.6	0.6
North	3	2	2.6	3.3	1.3	1.9	1.0	0.3	0.8	0.3	0.4	0.4
		4	4.6	2.3	1.6	1.1	1.0	0.4	0.8	0.6	0.5	0.4
	4	5	6.0	2.1	1.0	1.6	0.8	0.4	0.5	0.4	0.4	0.5
	7.3	7.8	6.0	2.6	2.2	1.5	1.2	0.7	0.7	0.7	0.6	0.6
North-Northeast	3	2	2.0	1.7	1.1	0.5	0.4	0.4	0.2	0.2	0.3	0.2
		4	3.9	1.4	0.8	0.7	0.5	0.6	0.3	0.3	0.2	0.3
	4	4	4.6	1.0	0.9	0.6	0.4	0.3	0.4	0.4	0.3	0.3
	7.3	7.8	8.0	2.1	1.7	1.3	0.9	0.7	0.6	0.5	0.6	0.5
Northeast	3	2	1.6	1.1	0.4	0.8	0.3	0.2	0.4	0.3	0.3	0.3
		4	4.1	1.1	0.9	0.7	0.5	0.3	0.4	0.3	0.3	0.3
	4	4	2.2	1.1	1.1	0.6	0.5	0.7	0.2	0.3	0.3	0.4
	7.3	7.8	5.5	1.7	1.7	1.1	0.8	0.7	0.6	0.5	0.5	0.5

Photo 1. Typical wave patterns at the canal entrance for the +5-ft swl; 4-sec, 4-ft test waves from northwest

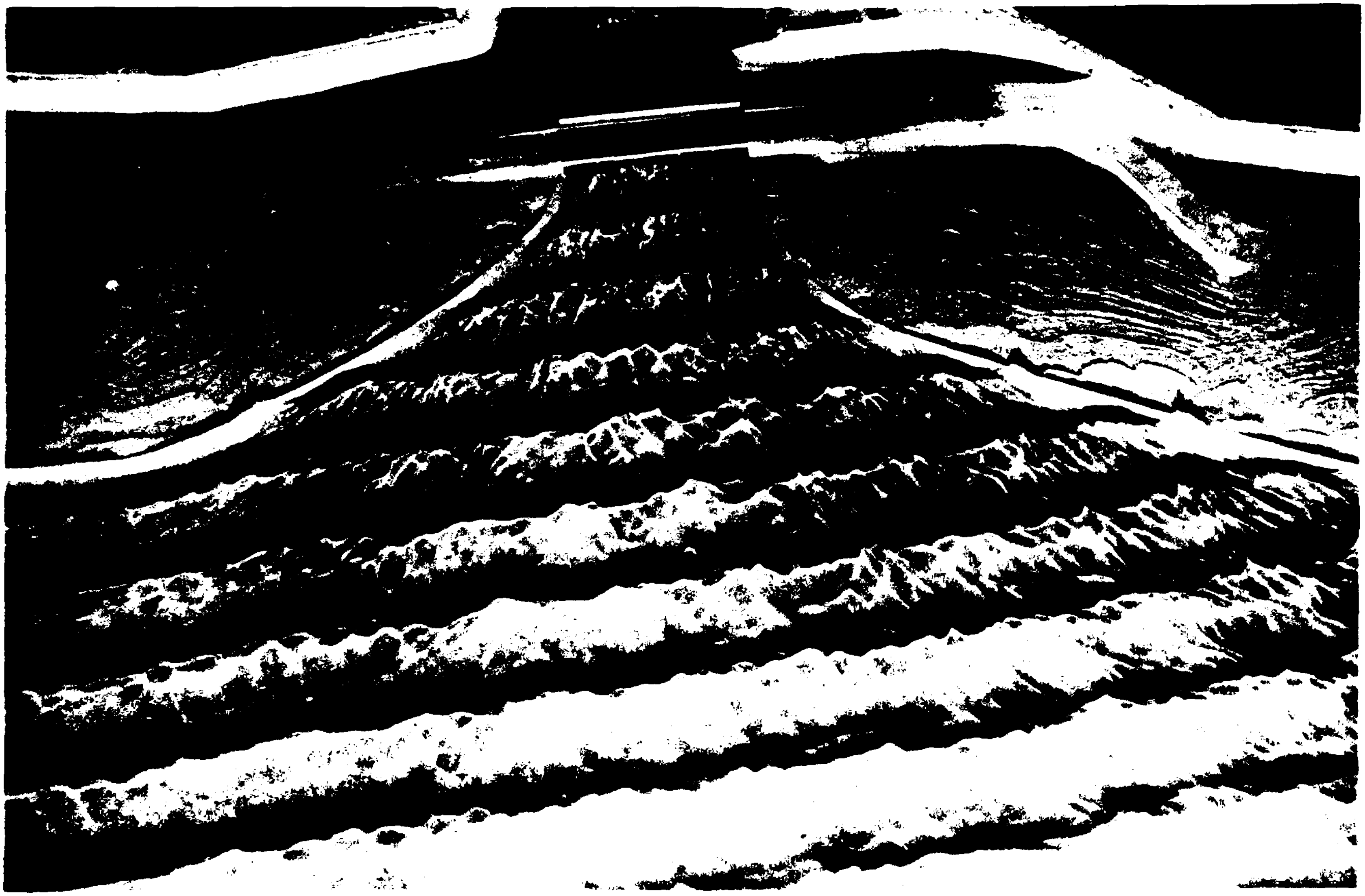

Photo 2. Typical wave patterns at the canal entrance for the +5-ft swl; 4-sec, 4-ft test waves from north-northwest

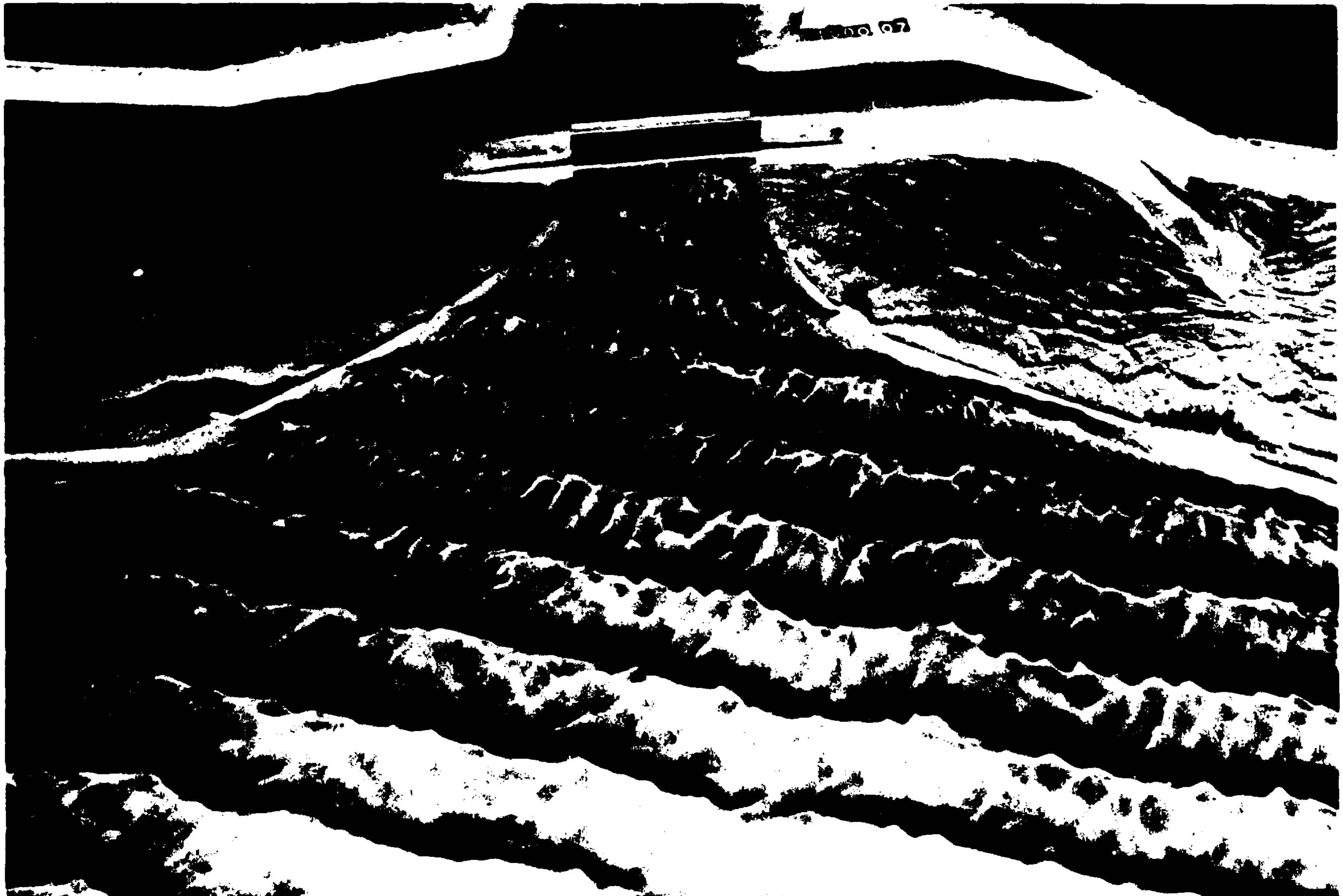

Photo 3. Typical wave patterns at the canal entrance for the +5-ft swl; 4-sec, 5-ft test waves from north

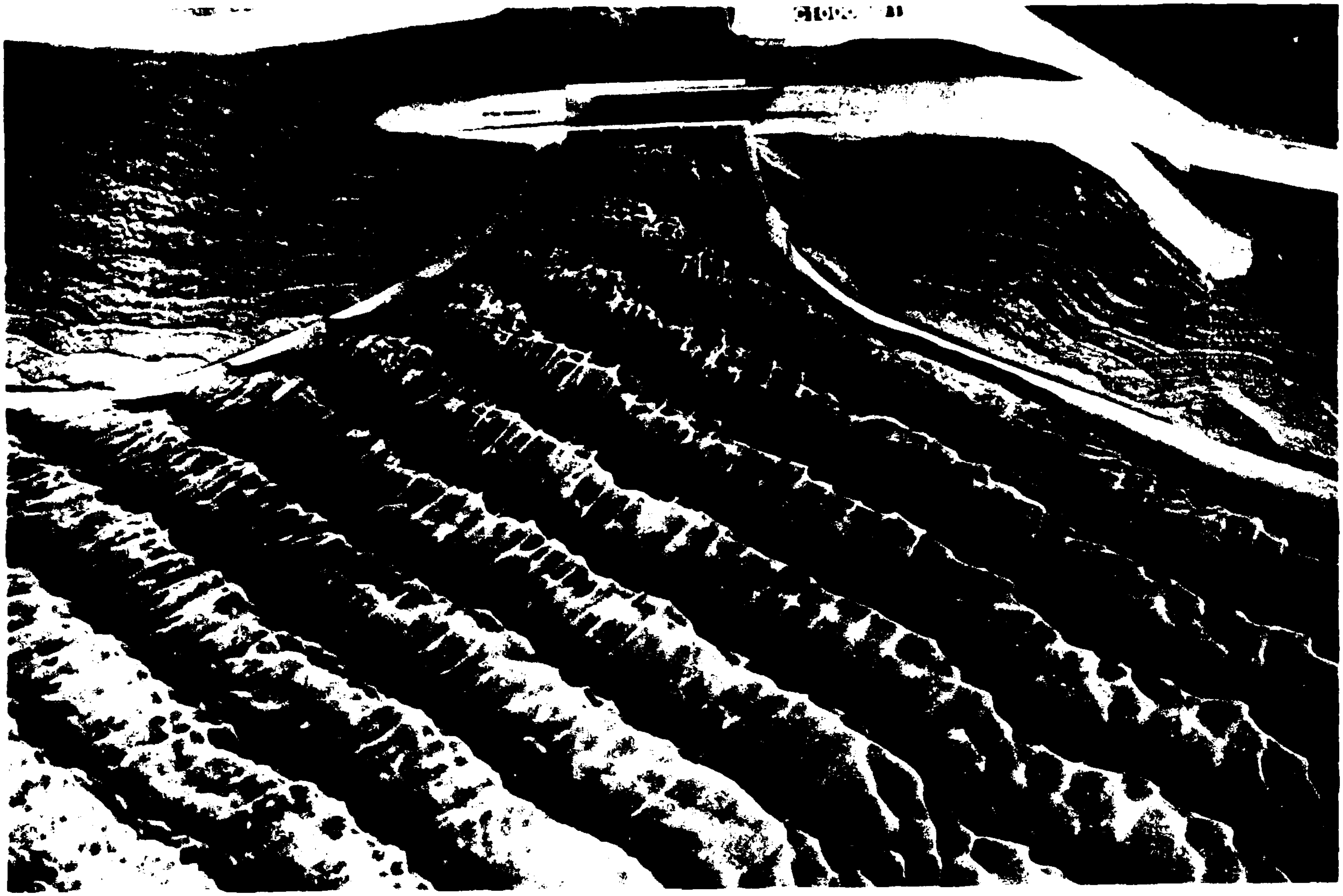

Photo 4. Typical wave patterns at the canal entrance for the +5-ft swl; 4-sec, 4-ft test waves from north-northeast

Photo 5. Typical wave patterns at the canal entrance for the +5-ft swl; 4-sec, 4-ft test waves from northeast

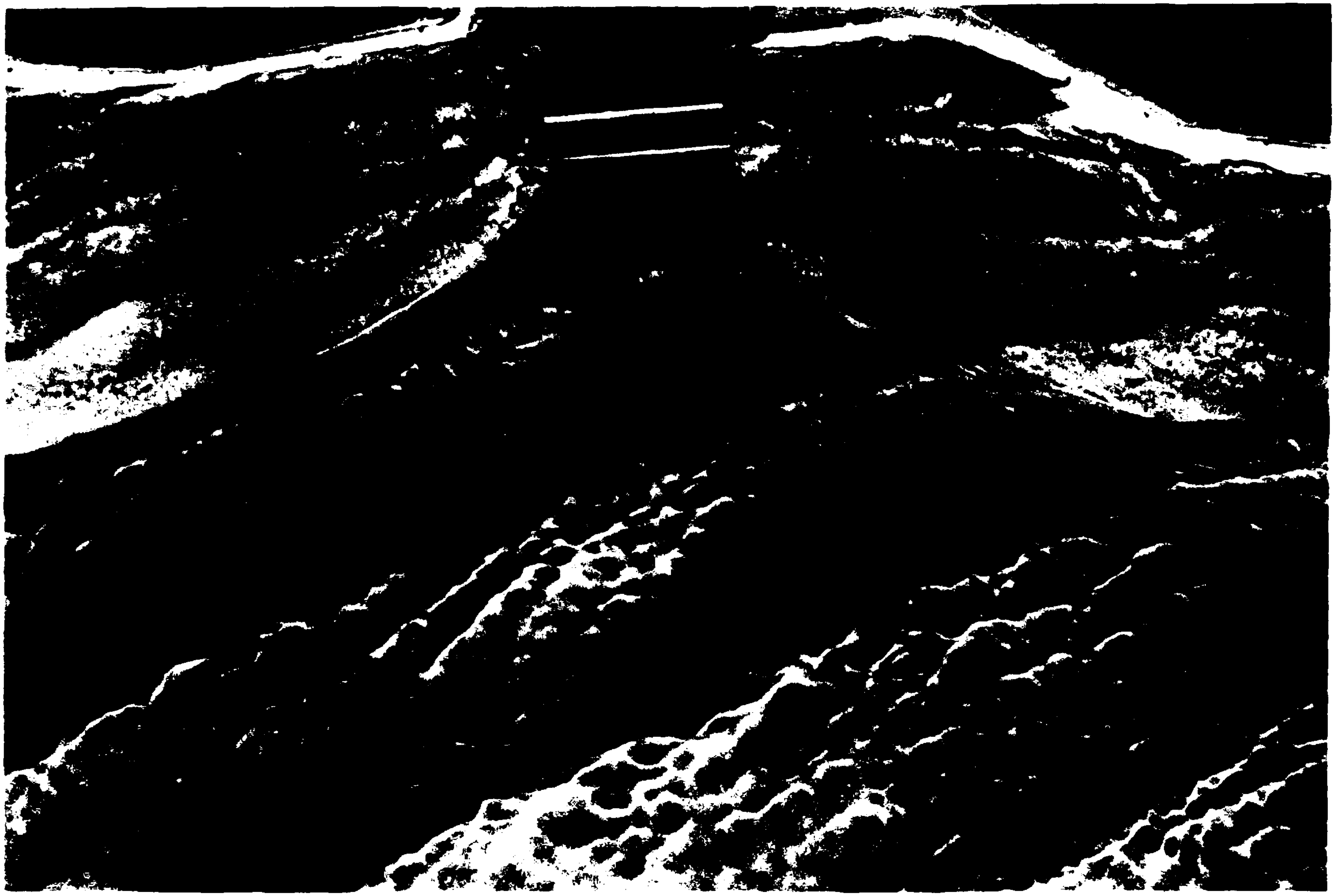

Photo 6. Typical wave patterns at the canal entrance for the +11.5-ft swl; 7.3-sec, 7.8-ft test waves from northwest

Photo 7. Typical wave patterns at the canal entrance for the +11.5-ft swl; 7.3-sec, 7.8-ft test waves from north-northwest

Photo 8. Typical wave patterns at the canal entrance for the +11.5-ft swl; 7.3-sec, 7.8-ft test waves from north

Photo 9. Typical wave patterns at the canal entrance for the +11.5-ft swl; 7.3-sec, 7.8-ft test waves from north-northeast

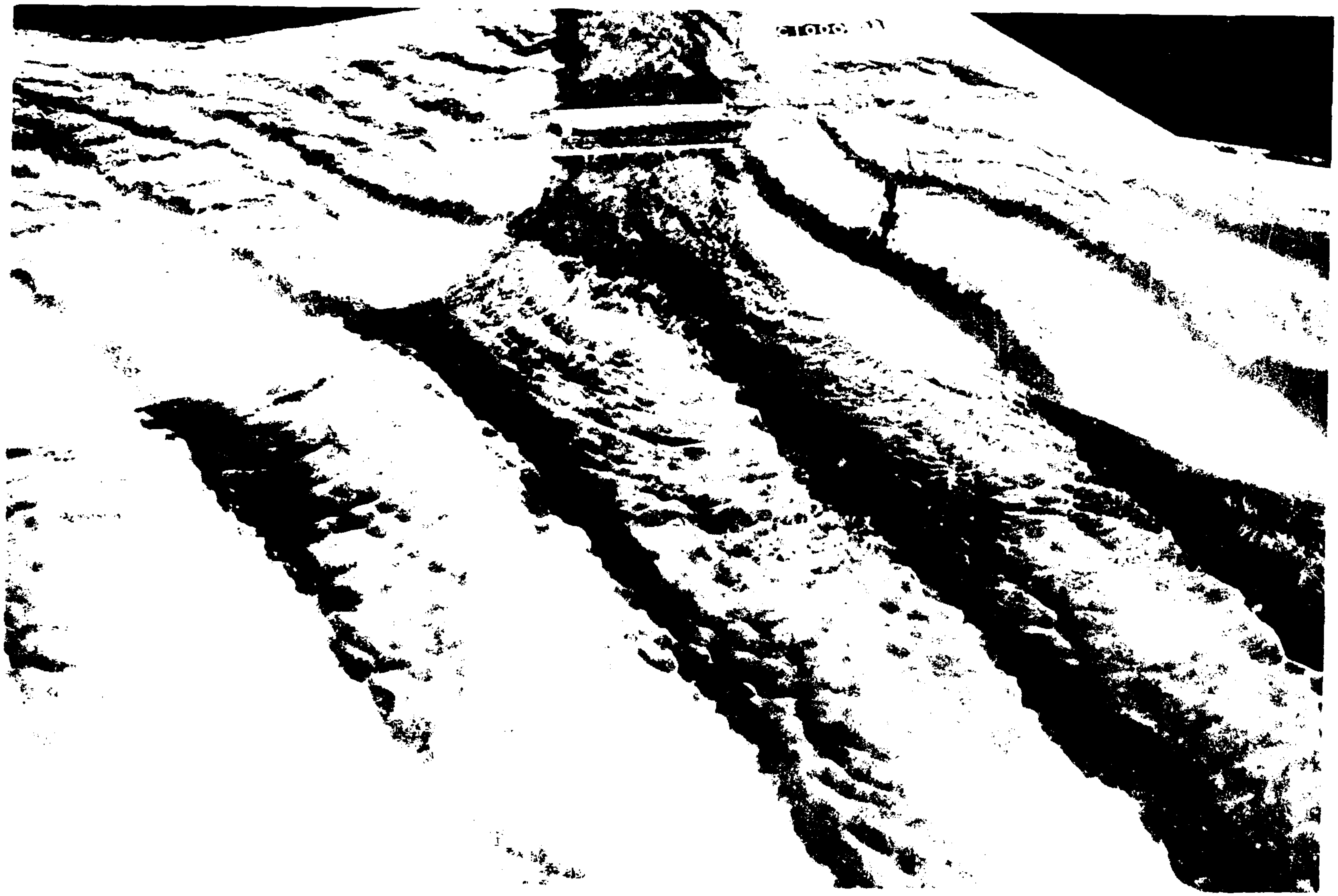

Photo 10. Typical wave patterns at the canal entrance for the +11.5-ft swl; 7.3-sec, 7.8-ft test waves from northeast

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